Physical Model of Current-Induced Scour at Ventura Harbor

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Abstract: This paper describes a 1:25-scale movable-bed physical model study of scour resulting from longshore current passing through a narrow gap on the leeside of the detached breakwater at Ventura Harbor, California, USA. The physical model was calibrated by adjusting the total flow discharge to achieve equilibrium scour development that matched the scour hole measured at Ventura Harbor. The calibrated model was then used to predict future scour potential and to optimize the design for remedial toe protection intended to prevent leeside armor layer damage on the detached breakwater. In addition, the model study identified and eliminated several construction problems that could have caused significant unforeseen expenditures.

Introduction

Since its original construction in 1963, the entrance to Ventura Harbor has undergone a series of engineering modifications in an effort to decrease deposition of littoral sediments in the navigation channel. Construction of a detached breakwater with a large sand trap in the lee eased the shoaling problem somewhat, but some sand still escaped the sand trap and entered the navigation channel. Further improvements alleviated the shoaling problem; however, a flow constriction was created that resulted in severe scour during a fierce storm. The scour undermined the leeside toe of the detached breakwater and caused the armor layer to slump.

This paper describes a movable-bed model investigation that established the scour mechanism, determined that future scouring of the breakwater toe was inevitable, and developed effective toe protection that resulted in significant cost savings. The model was calibrated by adjusting the flow condition to reproduce the original scour hole measured at Ventura Harbor. Simple analytical tools provide reasonable estimates of

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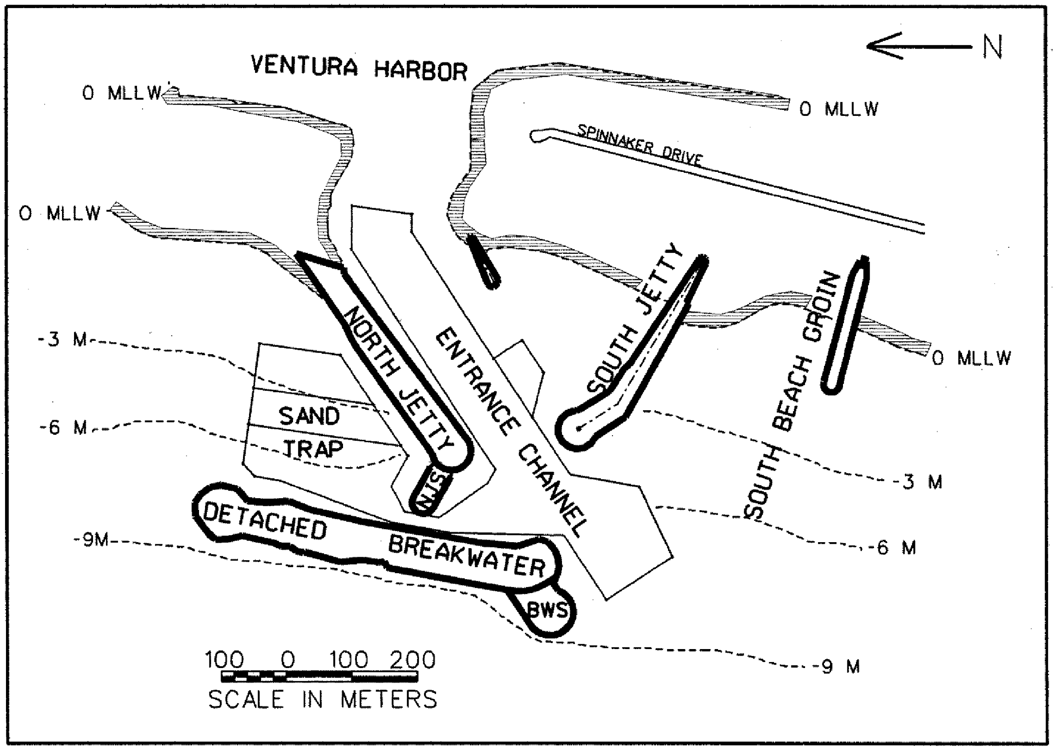

Figure 1: Harbor entrance at Ventura Harbor, California.

scour potential, but the movable-bed model was critical for verifying hypotheses and designing proper engineering solutions.

Background and History of Ventura Harbor

Ventura Harbor is a man-made commercial and recreational harbor located on the southern California coast approximately 100 km northwest of Los Angeles, California. Besides the approximately 200 commercial berths and 1,600 recreational berths, the harbor provides ocean access to an attached private marina containing 300 boat slips. The principal structural features of the present-day harbor entrance are two rubble-mound jetties, a beach groin to the south of the entrance, and a detached rubble-mound breakwater as shown in Figure 1.

Mean tide range at Ventura Harbor is 1.13 m, and the range between mean-lower-low water (MLLW) and mean-higher-high water (MHHW) is 1.65 m. Offshore islands limit wave approach to three relatively narrow corridors: west, southwest, and south. Ninety percent of the time waves approach from a westerly direction through the Santa Barbara Channel, creating a southerly drift of littoral material (Hughes and Schwichtenberg 1998).

Ventura Harbor is located within the Santa Barbara littoral cell between Ventura River to the north and the Santa Clara River to the south. A 1989 sediment budget estimated the annual average shoaling rate at Ventura Harbor to be about 490,000

m^3/yr with 413,000 m^3/yr of sediment arriving from the north and 77,000 m^3/yr moving up from the south (Schwichtenberg, et al. 1997).

Early History

Ventura Harbor was constructed by local interests in 1963, and the original design featured the arrowhead jetties, a middle groin, entrance channel, turning basin, and three berthing basins. Because of funding limitations, the arrowhead jetties were not constructed to full design length. This resulted in a wider entrance opening and jetties terminating in shallower water, factors that contributed to excessive channel shoaling, created dangerous wave conditions, and effectively closed the entrance an average of 66 days per year (Adams 1976).

1968 Improvements

In 1968 the US Army Corps of Engineers accepted responsibility for the entrance channel and navigation structures. The Corps constructed a 457-m-long detached breakwater with a large sand trap in the breakwater lee to the north of the north jetty (see Figure 1). The breakwater was intended to decrease wave heights so longshore moving sand would settle in the sand trap and so navigation in the entrance channel would be safer. The breakwater also provided shelter for dredging operations; however, with an original crest elevation of 4.3 m MLLW the detached breakwater is heavily overtopped during larger storms. The sand trap was excavated to depths ranging between -8 m to -12 m MLLW to give a capacity of about 612,000 m^3. It was anticipated that dredging would eventually occur on a two-year cycle. Construction of the detached breakwater and sand trap was completed in 1972.

The 1972 modifications were only partially effective. Rip currents and sand accumulation along the north jetty allowed sand to bypass a portion of the sand trap and deposit in the entrance channel, and annual maintenance dredging was required to maintain a -6 m MLLW entrance channel project depth. Between 1973 and 1989 dredging costs were US$15 million. Problematic shoaling in the entrance channel created dangerous navigation conditions; and between 1982 and 1990, there were 60 capsized or damaged vessels and 11 injuries at Ventura Harbor entrance. Hazardous conditions prevented vessels from navigating the entrance during a substantial portion of the year.

1994 Improvements

In 1989 the Corps of Engineers developed modifications to the Ventura Harbor structures and entrance channel to help alleviate channel shoaling and associated dangerous wave conditions. The selected plan included four elements:

1. Construction of a 91-m-long spur groin (labeled “NJS” on Figure 1) off the tip of the north jetty angled toward the sand trap. The purpose of this extension was to deflect longshore currents to create a clockwise gyre, thus promoting deposition for a more productive use of the sand trap.

2. Construction of a new South Beach rubble-mound groin 300 m south of the south jetty. This structure extended about 200 m offshore to a depth of -1.8 m

MLLW. The groin was intended to impound northward moving sediment before it reached the navigation channel.

3. Construction of a 91-m extension to the south end of the detached breakwater (labeled "BWS" on Figure 1) to provide improved wave protection for vessels and dredge equipment in the navigation channel.

4. Deepening of portions of the navigation channel from a depth of -6 m to a new depth of -12 m MLLW to provide sand storage volume for advanced maintenance.

Figure 1 shows the location and orientation of the improvements. The design of the new structures were optimized using a 1:75-scale fixed-bed physical model (Bottin 1991). The model indicated wave heights would be reduced in the entrance channel by the detached breakwater extension. The effectiveness of the north jetty spur and the south groin were examined using crushed coal tracer. Construction of the improvements began in 1993 and was completed in August, 1994. Total cost of the improvements was US$6.46 million.

Scour Concerns at Ventura Harbor

Scour Hole Formation

A few months after completion of the 1994 Ventura Harbor improvements, harsh winter storms out of the west impacted the project. In January, 1995, on-site inspection revealed damage to the detached breakwater leeside armor layer originating immediately across from the tip of the north jetty spur and extending southward. Approximately 46 lineal meters of the two-stone-thick armor layer experienced slumping above and below the water line, and the breakwater crest in this region was significantly lowered. A hydrographic survey indicated a large scour hole had formed in the gap between the north jetty spur and detached breakwater (see lower sketch of Figure 4). The portion of the scour hole adjacent to the detached breakwater roughly corresponded to the region of leeside armor damage. The scour hole had a maximum depth of -9.5 m MLLW, whereas before the storm depths were close to the MLLW elevation.

The scour was caused by the *"jetting"* action as water passed through the flow constriction formed by the gap between the detached breakwater and north jetty spur. Figure 2 shows the potential flow solution for an inviscid jet having the same structure geometry as the gap region at Ventura Harbor.

The contours running across the streamlines represent lines of constant discharge, and the contour labels correspond to values of dimensionless discharge per unit width given by

$$\frac{q_e}{(Q/a)} = \text{Constant} \tag{1}$$

where q_e is the discharge per unit width, Q is the total discharge, and a is the distance between the breakwater and the north jetty spur. The fraction Q/a can be interpreted as the *"average discharge per unit width."*

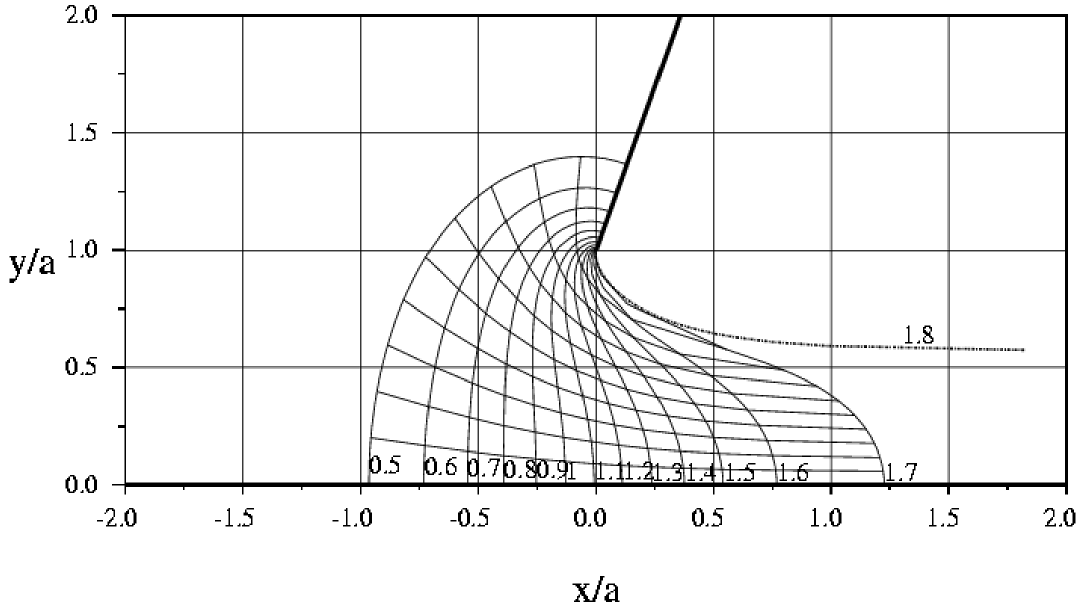

Figure 2: Potential flow map of jetting action through the gap.

As water flows through the gap the flow cross-section is constricted to almost half of the gap width. This causes the flow to accelerate rapidly until the discharge per unit width downstream of the gap reaches 1.7 times the "average discharge." Although the jet flow map is instructive, it does not include the effects of turbulent flow entrainment at the jet boundary that begins immediately downstream of the gap. Flow entrainment reduces the flow velocities and spreads the discharge distribution as the jet progresses downstream as illustrated in Figure 3. Nevertheless, estimates from potential jet flow theory are reasonable in the region less than one or two "gap widths" downstream of the gap.

It was concluded that the principal cause of damage to the detached breakwater leeside armor layer was due to jet-induced scouring of the breakwater toe and subsequent slumping of the armor layer. The -6 m MLLW contour of the scour hole intruded along the detached breakwater toe a distance of approximately 55 m (see Figure 4). This undermined the breakwater toe which was constructed at -3.7 m MLLW. Wave overtopping may have contributed to the breakwater leeside armor damage; but overtopping can ruled out as the principal cause of damage because the actual damage was confined to just one area, whereas overtopping damage would be expected to occur in isolated pockets along the entire breakwater length. The north jetty spur experienced no toe damage due to the scour hole because the toe was constructed at a lower elevation (-4.2 to -5.2 m MLLW), and the jetty spur toe was lined with 1-tonne stone left over from the original spur jetty construction.

Emergency Repairs

The Corps of Engineers undertook emergency repairs that included: dredging, construction of a rock sill along the gap between the north jetty spur and detached breakwater, and reconstruction of 61 lineal meters of the damaged breakwater. The sill was constructed by infilling the scour hole with "C-stone" (quarryrun stone having 0.08 to 0.5 m diameter) to a depth of -6.5 m MLLW followed by a 2-m-thick cover layer of "B-2 stone" (1.8-tonne stone) to give a final sill elevation of -4.5 m MLLW. The sill placement is shown on Figure 3. Repairs were completed by May 1995 for a total estimated cost of US$1.16 million as detailed in Hughes and Schwichtenberg (1998).

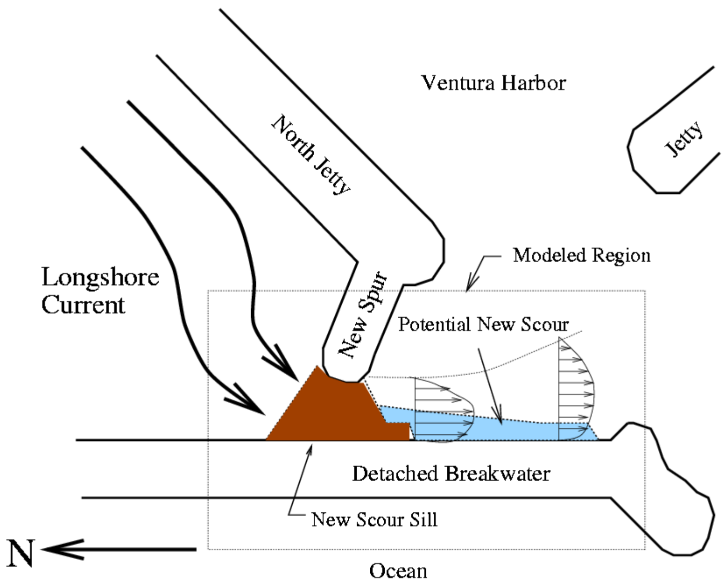

Figure 3: Scour sill and potential toe scour region.

Future Scour Concerns

Placement of the rock sill effectively prevents future scour in the immediate vicinity of the gap; but the sill also restricts the cross-sectional area of the gap, resulting in the potential for increased flow velocities over the sill and farther downstream. Past observations indicated that during normal conditions southerly flow through the gap between the north jetty spur and detached breakwater would not be strong enough to cause significant scour downstream of the rock sill. However, there was concern that during storm conditions a powerful jet would form, causing scour and undermining of the unprotected detached breakwater toe downstream of the sill. This could lead to damage of the armor layer on the leeside slope and additional costly repairs. Figure 3 illustrates the potential scour region downstream of the new scour sill.

An estimate of the potential increase of flow velocity through the gap due to the new sill can be calculated using a simple formulation for equilibrium discharge per unit width. By assuming the bottom stress caused by a fully turbulent boundary layer is proportional to the critical shear stress of noncohesive sediment during live-bed equilibrium, Hughes (1999) derived a formula for the maximum discharge per unit width. The maximum equilibrium discharge is the maximum flow rate that can be maintained for a given water depth without causing additional scour. An empirical constant in the formula was established using data from two tidal inlets. (See the Appendix of this paper for a summary of the development).

The maximum equilibrium discharge per unit width that occurred during formation of the original scour hole formation is estimated by substituting the maximum scour depth (9.5 m) and the sediment grain-size (0.2 mm = 0.0002 m) into Eqn. 10 in the Appendix, i.e.,

$$q_e = 5.12 \left[(9.807 \, m/s^2) \, (2.65 - 1) \right]^{1/2} \, (0.0002 \, m)^{\,3/8} \, (9.5 \, m)^{9/8} = \underline{10.6 \, m^2/s}$$

This estimate of q_e can be substituted into Eqn. 1, along with the gap width of $a \approx 38 \, m$, to estimate the total discharge as $Q \approx 400 \, m^3/s$. The *mean velocity* corresponding to the maximum equilibrium discharge per unit width is found to be

$$\bar{V}_{scour} = \frac{q_e}{h_{max}} = \frac{10.6 \, m^2/s}{9.5 \, m} = \underline{1.1 \, m/s}$$

This estimate assumes that the scour hole had reached maximum depth, and that no infilling had occurred between the scour event and subsequent survey.

Placement of the rock sill reduced the maximum depth in the gap to just 4.5 m. Therefore, the mean velocity over the sill for a storm that produces the same maximum discharge will be

$$\bar{V}_{sill} = \frac{q_e}{h_{max}} = \frac{10.6 \, m^2/s}{4.5 \, m} = \underline{2.4 \, m/s}$$

Velocities this high are expected to carry across the sill, and erode the sand adjacent to the detached breakwater toe.

Due to the uncertainty of the scour potential, the Los Angeles District of the Corps of Engineers planned to install toe protection along selected regions of the breakwater south of the rock sill. The initial toe protection design was based on engineering judgment and past experience, and this design was tested and refined in movable-bed laboratory tests conducted at the Waterways Experiment Station during April–June, 1997.

Laboratory Study

The movable-bed physical model study was conducted with two primary objectives:

1. Determine if flow conditions similar to the flow that caused the original scour hole were likely to cause additional scour downstream of the rock sill, thus endangering the detached breakwater leeside armor. The outcome of this test would support the decision on whether or not to fund construction of a protective toe berm along the leeside toe of the detached breakwater.

2. Test and refine the 1996 provisional toe berm design if the initial model results indicated that a toe berm is needed to protect the leeside armor layer from undermining.

The region of Ventura Harbor entrance shown by the dashed line on Figure 3 was constructed in a large wave facility at the Waterways Experiment Station in Vicksburg, Mississippi. The model was constructed at a prototype-to-model length scale of $N_L = 25$ on a flat-bottom portion of the flume that features a rectangular movable-bed section. Flows representing longshore currents were generated by pumping water through a wide current manifold, and vertical guide walls helped direct the flow toward the gap between the north jetty spur and detached breakwater. Additional description of the physical model configuration and operation is given in Hughes and Schwichtenberg (1998).

Movable-Bed Modeling Criterion

The main difficulty with movable-bed models is obtaining correct similitude between the prototype and model sediments. Ideally, for situations were sediment is moving primarily by bedload, the model sediment should scale the same as the length scale, whereas suspended sediment transport appears to scale by sediment fall speed (Hughes 1993). The mean sediment grain-size diameter in the Ventura Harbor area is about 0.19 mm, and the model sand had a mean diameter of 0.13 mm, giving a prototype-to-model sediment diameter scale of $N_{d_e} = 1.46$. Thus, model similarity by strict similitude considerations for either bedload or suspended sediment transport was impossible.

An alternative method for achieving model similarity with the prototype is to reproduce a distinctive characteristic of the physical process. For the case of equilibrium scour caused by steady flow, a logical choice is to maintain similarity of the equilibrium discharge relationship that balances the flow boundary layer with the critical shear stress of the sediment.

First rearrange the form of the equilibrium discharge relationship given by Eqn. 12 in the Appendix into

$$\frac{\bar{V}}{(g\gamma_i)^{1/2}\, d_e^{3/8}\, h_e^{1/8}} = Constant \tag{2}$$

where

$$\gamma_i = \left(\frac{\rho_s - \rho_w}{\rho_w}\right) \tag{3}$$

Similarity requires that the constant on the right-hand side of the Eqn. 2 must be the same in the model as in the prototype. Taking the prototype-to-model ratio of Eqn. 2, and expressing the result in terms of scale factors yields

$$N_{\bar{V}} = (N_g N_{\gamma_i})^{1/2}\, N_{d_e}^{3/8}\, N_z^{1/8} \tag{4}$$

This scaling criterion is identical to that proposed by Kamphuis (1975) and derived by similar boundary layer considerations.

Applying this scaling criterion to the Ventura Harbor physical model with $N_z = N_L = 25$, $N_{d_e} = 1.46$, and $N_g = N_{\gamma_i} = 1$ gives a velocity scale of

$$N_{\bar{V}} = \frac{V_{proto}}{V_{model}} = (1)^{1/2}(1.46)^{3/8}(25)^{1/8} = \underline{1.72}$$

In other words, to reproduce the scour hole formed in the prototype under an estimated mean velocity of 1.1 m/s, the model would need a mean velocity of

$$\bar{V}_{model} = \frac{\bar{V}_{proto}}{1.72} = \frac{1.1\,m/s}{1.72} = \underline{0.64\,m/s}$$

Note that strict Froude scaling of the velocity would have yielded a model velocity of 0.22 m/s, which is beneath the velocity required for incipient motion of the model sediment. Therefore, the model scaling given by Eqn. 4 distorts the Froude velocity to achieve similarity of the maximum scour depth. We note that the discussion of scaling originally given in Hughes and Schwichtenberg (1998) is incorrect.

Model Calibration

There was enough uncertainty surrounding the proposed movable-bed scaling criterion and estimate of total discharge that occurred during the scour hole formation, that was necessary to *"calibrate"* the movable-bed model by attempting to reproduce the original Ventura scour hole in the physical model. Calibration of the physical model was an essential component of this study if any credence were to be given to the model scour results.

Calibration was performed by incrementally increasing the total discharge through the gap and comparing the equilibrium scour depth to the actual target depth. This procedure follows the philosophy of distorting the velocity scale to achieve similarity. Eventually, the equilibrium scour depth and general planform of the scour in the model approximately matched the prototype at which time the bathymetry was recorded, contoured and plotted. Two calibration tests were performed without the rock sill in place across the gap. The first test served as a trial run, and the second test was considered the calibration run. The second calibration test lasted 14 hours, and produced a scour pattern quite similar to the prototype scour. Figure 4 compares model and prototype scour hole contours. The plots are reproduced at approximately the same scale, and contour labels refer to prototype depth in meters. Additional details are given in Hughes and Schwichtenberg (1998).

Scour After Rock Sill Construction

The rock sill spanning the gap between the north jetty spur and detached breakwater at Ventura was constructed to scale in the physical model with the sill crest elevation corresponding to a prototype elevation of -4.5 m MLLW. This test configuration represented the existing condition at Ventura, and the test was conducted to determine the likelihood of scour developing downstream of the rock sill under conditions similar to what caused the original scour hole. Model water depth was once again 18 cm (water surface corresponding to MLLW in prototype), and currents were generated using the same flow rate as was used in the second calibration test. Scour equilibrium was reached after 12 hours, and the bed was surveyed and photographed.

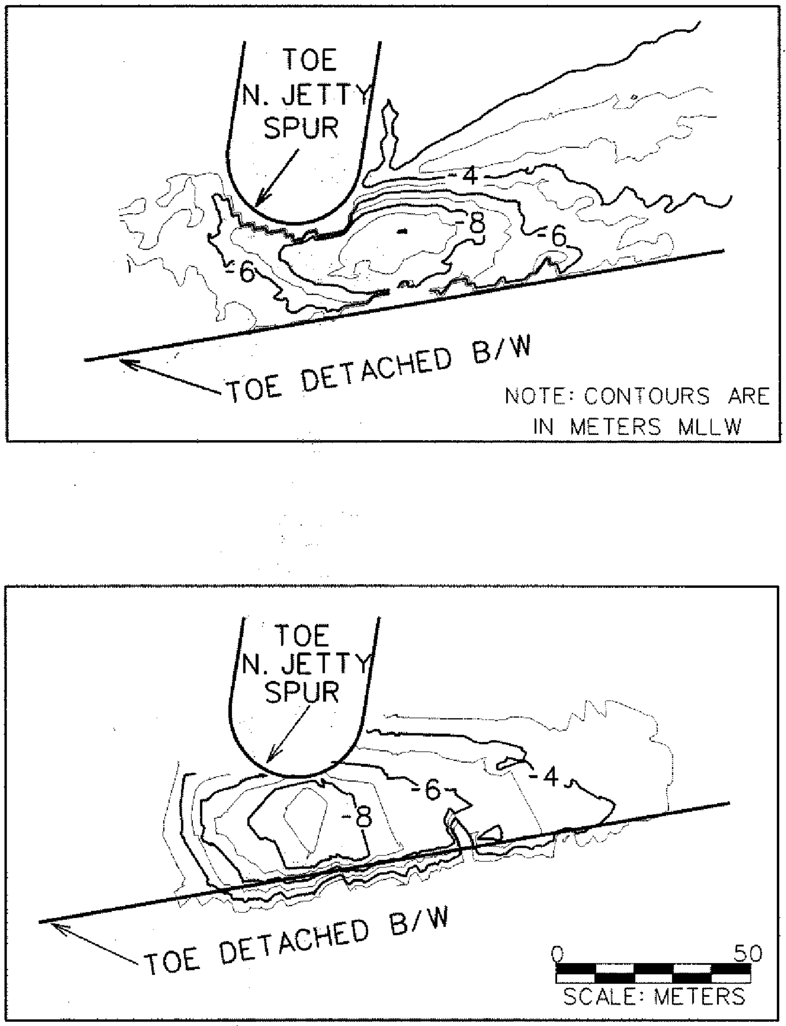

Figure 4: Model (upper) and prototype (lower) scour hole contours.

Scour contours measured in the model downstream of the rock sill are shown in the sketch of Figure 5. Contour values correspond to prototype depths in meters. The scour hole attained a depth greater than the scour that occurred in the second calibration test as shown in the upper sketch of Figure 4. Besides being displaced downstream of the sill, the scour hole was narrower and more elongated than scour that occurred without the rock sill in place. Where the scour hole impinged on the breakwater toe, armor stones became unstable and fell into the scour hole.

The model test of the existing Ventura configuration strongly supported the hypothesis that current-induced scour at Ventura Harbor had the potential to undermine the leeside armor slope along the entire length of the detached breakwater south of the rock sill. This would cause considerable damage to the leeside armor slope, and expensive repairs would be needed. Therefore, construction of a protective toe berm along the detached breakwater was warranted. In the following discussion, all stated values are in prototype units.

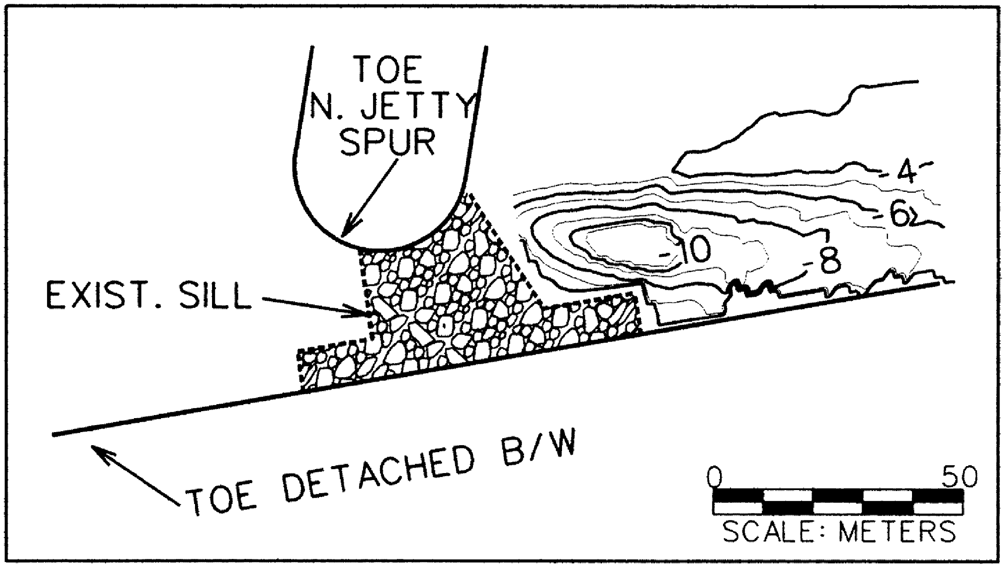

Figure 5: Ventura Harbor model scour downstream of rock sill.

Design of Toe Protection

The 1996 design for toe protection along the leeside of the detached breakwater was prepared before the investigation described in this paper was undertaken. This early design featured 5-m-wide berm of B-2 (1.8 tonne) stone at elevation -1.2 m MLLW and an additional 10-m-wide apron of quarryrun C-stone at an elevation of about -4 m MLLW. However, only 46 lineal meters of the 136 lineal meters of breakwater downsteam of the rock sill was to receive new toe protection.

Results from the model test with sill in place (but with no toe protection) indicated that the unprotected 92 lineal meters of breakwater would be vulnerable to scour-related damage under the 1996 toe protection design. Therefore, future repairs could be avoided if the toe protection was extended over the entire 136 m of breakwater. It was also felt that the 1996 design cross-section was overly conservative, and cost savings could be realized by optimizing the design using the physical model. A total of three design options were tested in the physical model, with each test improving on some aspect of the previous design.

Plan 1. The first plan featured a 7.6-m-wide, 1.8-m-deep berm constructed of quarryrun material (0.16 m to 0.3 m stone diameter) at an elevation of -2.8 m. The model test was run for 9 hours to scour equilibrium at the same flow rate used in the second calibration test. Next, opposing waves were added to simulate severe waves diffracting around the southern tip of the detached breakwater and approaching the gap from the south.

The scaled quarryrun C-stone proved to be stable under current and wave action. In fact, the result was conservative because the currents were relatively faster than called for by Froude scaling, which governed toe berm stability. Overall, a significant portion of the berm bench slumped into the scour hole that developed in the model, thus forming a protective slope. This left between 3 m to 6 m of original berm undisturbed.

Plan 2. One worrisome aspect of Plan 1, and the 1996 design, was the need to excavate to a depth below the toe of the 1992-repaired armor slope (located at -3.7 m MLLW) to place the new toe berm. During excavation there would be a risk of initiating a slope failure, resulting in costly repairs and project delay. Plan 2 was designed with a 1-m-thick layer of B-2 stone ranging between 0.63–0.95 m in stone diameter, over a 1-m-thick layer of C-stone quarryrun. This plan featured a 3-m-wide horizontal bench at -1.0 m elevation and a sloping berm having a 14.3-m horizontal extent. At the end of the test, a scour trench had formed and a portion of the sloping berm was undermined, but Plan 2 provided plenty of reserve protection, and the design should protect the structure for currents well above the flow condition tested.

The Los Angeles District of the Corps of Engineers planned to construct the protective toe berm as part of a maintenance dredging contract that was being put out to bid at about the same time as the Plan 2 test was completed. Based on physical model results, the Plan 2 design replaced the 1996 design in the bidding documents. However, after bids were requested, it was realized that the berm elevation of Plan 2 was significantly higher than the -4.5 m elevation of the rock sill (also a problem with the 1996 design). This would prevent larger dredges from accessing the sand trap to the north of the north jetty, and it represented a serious navigation hazard for smaller dredges and other vessels. Potential costs associated with dredge and vessel grounding, and possibly the need to remove the berm at some later date, made Plan 2 much less desirable than originally thought. (Additional details and cross-sections for the Plan 1 and Plan 2 toe protection designs are given in Hughes and Schwichtenberg (1998).

Plan 3. The potential problems of Plan 2 were corrected by the Plan 3 design shown in the upper portion of Figure 6. This berm was composed of only C-stone quarryrun and featured an 11.6-m-wide by 3-m-thick horizontal bench at an elevation of -4 m MLLW. Risk to the existing breakwater toe was lessened by excavating on a slope as illustrated in Figure 6.

The model was run for 9 hours under the same water depth and flow conditions as the previous two alternative plan tests. Plan 3 proved to be fully adequate as indicated by the lower sketch of Figure 6. Consequently, the Los Angeles District issued an amendment to the plans and specifications utilizing the toe berm design shown in the Plan 3 cross-section.

Summary and Conclusions

Since its original construction in 1963, the entrance to Ventura Harbor has undergone a series of engineering modifications in an effort to decrease deposition of littoral sediments in the navigation channel. In 1994 a spur groin was added to the tip of the north Ventura Harbor jetty, narrowing the gap between the north jetty and detached breakwater. This modification appeared to promote increased trapping of sediment during normal weather conditions; but during strong storms, southward

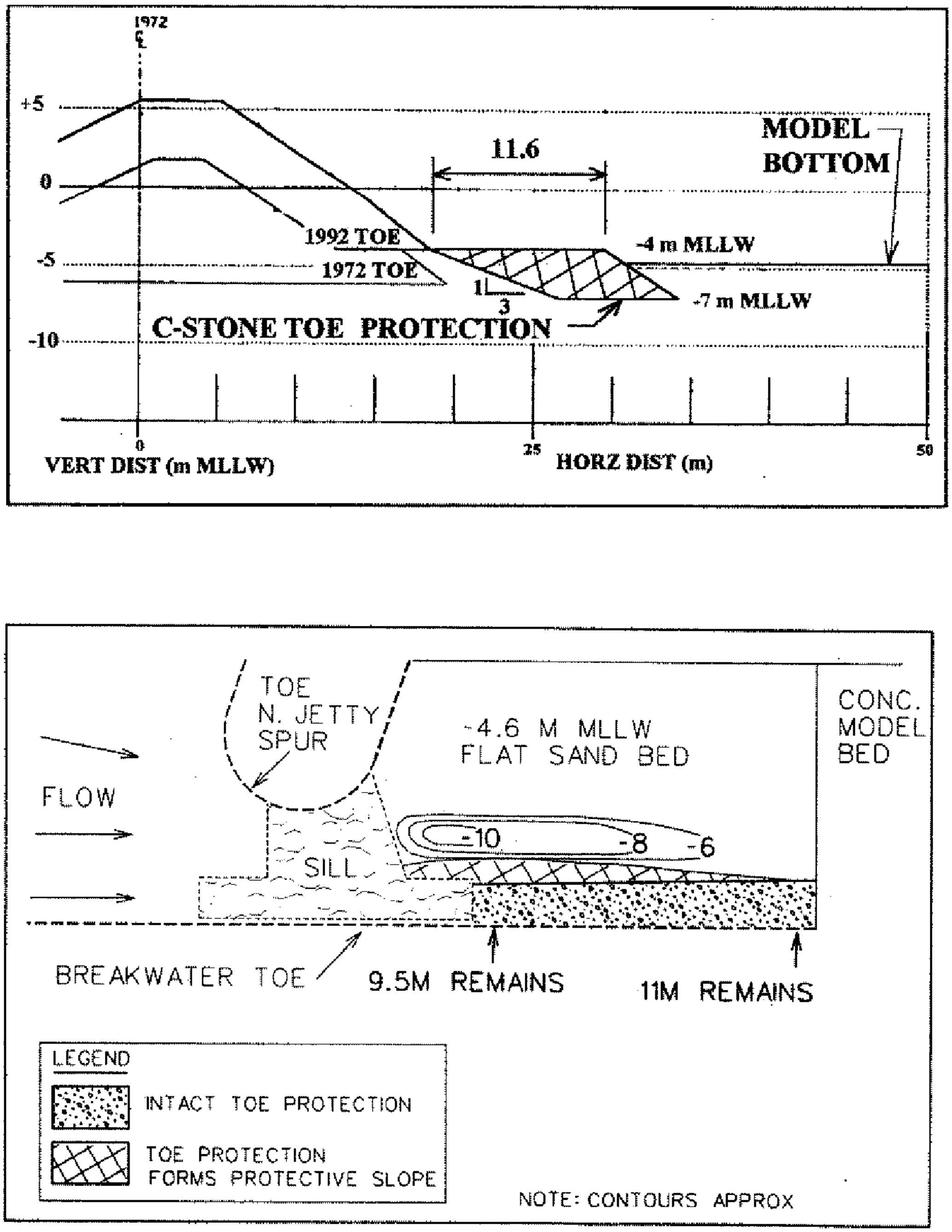

Figure 6: Plan 3 detached breakwater toe protection.

flowing longshore currents accelerated through the narrow gap in a manner similar to a jet. This resulted in high velocities that created a large scour hole over 9 m deep. After an emergency repair to the detached breakwater and filling of the scour hole with rock, there was concern that scour would continue to occur downstream of the repaired scour hole. This concern was strengthened by a simple estimate of the flow rate that caused the scour and by evaluation of the consequences of the same flow rate passing over the rock sill.

A 1:25-scale laboratory movable-bed model of the Ventura Harbor scour region was built to determine the probable extent of scour that could be expected, and to test and refine the 1996 scour protection design to assure adequate protection from future scour. The model was calibrated by adjusting the total flow discharge to achieve equilibrium scour development in the model that reasonably matched the scour hole measured at Ventura. Although the model similitude was attained via calibration, it appears that the similarity criterion developed from the equation for maximum equilibrium discharge would be appropriate in cases where validation data are not available. The Ventura Harbor model study was completed successfully, and the following benefits were realized as a direct result of the study:

- The model study confirmed that scour would indeed undermine the detached breakwater toe. Thus, the proposed toe reinforcement was shown to be necessary, and the toe protection needed to extend over the entire toe.

- The physical model was used to optimize the toe apron cross-section design and to demonstrate the optimized design was stable with sufficient residual protection. The final design cross-section used 25 percent less stone than the 1996 design.

- While working in the physical model it was realized that the original 1996 toe protection design and the Plan 2 design were to be constructed at an elevation that would hinder passage of the dredge through the gap between the detached breakwater and the north jetty spur. The final design corrects this error, thus avoiding potential costs related to grounding of the dredge and subsequent rebuilding of the toe protection to correct the problem.

- The 1996 toe protection design called for excavation that involved a risk of leeside armor layer instability, whereas the final design is easier to construct with less risk to the existing structure.

Shortly after completion of the toe protection in 1997, three large storms impacted the project area. Visual field observations during these conditions support the existence of strong southerly flow through the gap. Estimates of surface current velocity were made by timing floating objects as they passed between two points of known separation distance. Flow velocity along the Ventura north jetty was estimated to be 1.5 m/s. Through the gap the current accelerated to values exceeding 2 m/s, which corresponds well to the estimate made based on the equilibrium discharge formulation. It was also noted that the currents "pulsated" in tune with the surf beat.

Acknowledgements

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Appendix: Equilibrium Discharge Relationship

Formulation: Assume the vertical velocity profile during maximum discharge through a tidal inlet can be represented as a steady, fully-developed, rough turbulent boundary layer extending from the bottom to the free surface. Any contribution by waves is neglected. The boundary layer velocity profile can be adequately approximated by a 1/8-power curve (Yalin 1971) with the shear stress at the bed given as

$$\tau_o = \rho_w \left[\frac{\bar{V}}{C_k \, (h/d_e)^{1/8}} \right]^2 \tag{5}$$

where

ρ_w	–	mass density of water
$\bar{V}$	–	depth-averaged velocity
C_k	–	undetermined constant
h	–	water depth at maximum discharge
d_e	–	median grain-size diameter

The constant C_k is a boundary layer shape factor that includes the unknown relationship between d_e and bottom roughness.

The *Critical Shear Stress* of the noncohesive sand bed is given by the Shields parameter as

$$\tau_{cr} = C_s \, (\rho_s - \rho_w) \, g \, d_e \tag{6}$$

with

C_s	–	constant of proportionality
ρ_s	–	mass density of sand
g	–	gravitational acceleration
d_e	–	median grain-size diameter

For live-bed equilibrium, a shear stress balance is assumed with $\tau_o \sim \tau_{cr}$. Equating Eqns. 5 and 6 results in the expression

$$\frac{h}{d_e} = \frac{1}{(C_e)^8} \left[\left(\frac{\rho_w}{\rho_s - \rho_w} \right) \left(\frac{\bar{V}^2}{g \, d_e} \right) \right]^4 \tag{7}$$

where the two unknown constants, C_k and C_s, have been combined into C_e. The term in square brackets on the right-hand side of Eqn. 7 is the ratio of grain-size Froude number to the immersed specific gravity of the sand, and it is defined as the *Grain Mobility Number* (Yalin 1971).

A more useful form of Eqn. 7 is obtained by multiplying both sides by h^8 and rearranging to get an expression for the equilibrium discharge per unit width, i.e.,

$$q_e = C_e \, [g \, (S_s - 1)]^{1/2} \; d_e^{3/8} \, h^{9/8} \tag{8}$$

where the q_e is defined as the **Equilibrium Maximum Discharge per unit width**, given by

$$q_e = \bar{V} h \tag{9}$$

and $S_s = \rho_s/\rho_w$ is the sediment specific gravity (about 2.65 for quartz sand). As expected Eqn. 8 indicates that the equilibrium maximum discharge is primarily a function of water depth with sediment size having a relatively minor effect.

Empirical Coefficient: The unknown coefficient in Eqn. 8 was empirically evaluated by comparison to field measurements at two dual-jetty tidal inlets. Vertical profiles of horizontal velocity were measured along transects at Shinnecock Inlet, New York, and at Ponce de Leon Inlet, Florida using a boat-mounted acoustic Doppler current profiler. Discharge per unit width was estimated from the measurements by integrating the velocity profiles over the depth. Profiling transects across the inlet throats occurred at or around the maximum ebb or flood flow.

The results are shown on Figure 7 where calculated discharge per unit width is plotted versus the term ($[g\,(S_s-1)]^{1/2}\ d_e^{3/8}\ h^{9/8}$) on the right-hand side of Eqn. 8. Grain-size for the Shinnecock Inlet channel was taken as 0.6 mm, whereas a size of 0.21 mm was used for Ponce de Leon Inlet. Both sands were assumed to have the same density as quartz.

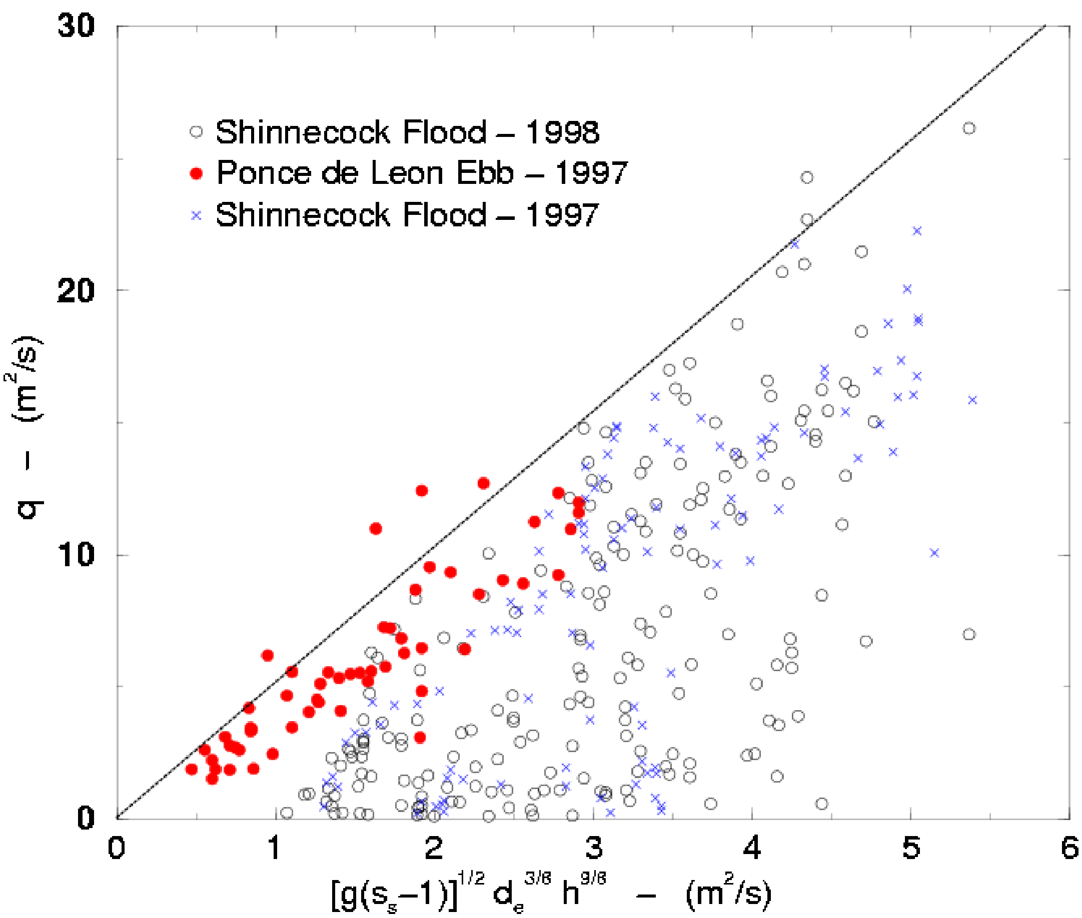

Figure 7: Field Data From Two Dual-Jettied Inlets.

The data points on Figure 7 show a wide range of discharge per unit width measured at the different depths. However, there is an upper limit to the data as indicated by the straight dashed line. This dashed line represents the maximum discharge per unit width (q_e) that can be sustained at a particular value of the parameter ($[g\,(S_s-1)]^{1/2}\ d_e^{3/8}\ h^{9/8}$). The discharge indicated by the dashed line is termed the **equilibrium maximum discharge**. Any increase in discharge beyond the equilibrium value will result in an increase in water depth.

The scatter of measurements beneath the dashed line is pronounced, and this indicates that the discharge calculated for those measurements was less than could be tolerated by the depth at that location. Points just beneath the dashed line might be locations where the present bottom was eroded by discharges slightly greater than those measured during the field exercises. Many of the data points well below the line came from inlet cross-sections either slightly seaward of the jetties where depths are controlled by waves and longshore currents; or landward of the entrance channel where the tidal current is insufficient to scour the channel, and depths have been increased by dredging.

Another explanation for data scatter below the dashed line is that depths at some of the locations are scoured by a different cross-channel flow distribution that occurs during the reverse maximum tidal flow. Finally, there is the possibility that some of the depths are the result of scouring that occurred during episodic events such as storm surges or river discharge combined with ebb flow. Regardless of the reason, depths associated with data points below the dashed line are not in equilibrium with the measured discharge. In other words these depths would be able to accommodate increased flow discharge without additional scouring of the bottom.

The dashed line in Figure 7 corresponds to $C_e = 5.12$ in Eqn. 8, which can now be expressed as an empirical equation for equilibrium maximum discharge per unit width, i.e.,

$$q_e = 5.12\,[g\,(S_s - 1)]^{1/2}\ d_e^{3/8}\,h^{9/8} \tag{10}$$

For a given noncohesive sediment there is an **equilibrium scour depth**, h_e, associated with the equilibrium discharge q_e. The depth h_e is taken relative to the tide level at maximum discharge. An expression for h_e is obtained by rearranging Eqn. 10 to get

$$h_e = \frac{0.234\,q_e^{8/9}}{[g(S_s - 1)]^{4/9}\ d_e^{1/3}} \tag{11}$$

Although it might be possible to have depths greater than the equilibrium scour depth, these depths would have to be caused by some process other than the maximum discharge at that location. Estimates of equilibrium scour depth from Eqn. 11 should be considered conservative because the estimates represent the outer envelope of the field data. In reality the maximum discharge per unit width may not persist long enough to allow scoured depths to reach the predicted equilibrium depth.

Finally, substitution of the value of C_e into Eqn. 7 and rearranging provides a relationship for mean velocity at a location in terms of the equilibrium depth and sand parameters, i.e.,

$$\bar{V} = 5.12\,[g\,(S_s - 1)]^{1/2}\ d_e^{3/8}\,h_e^{1/8} \tag{12}$$

Physical Model of Current-Induced Scour at Ventura Harbor

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Keywords

- detached breakwater
- jet flow
- laboratory model
- model calibration
- movable-bed modeling
- scour
- similitude
- toe protection design
- Ventura Harbor